TECHNICAL NOTE

FAILURE PROCESS OF STONE COLUMNS IN COLLAPSIBLE SOILS

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Abstract Many countries in the word have loose and unstable soils encompassing a wide range of geological materials, which when inundated, may collapse and cause very significant distress to the structure. Stone columns have been used to strengthen such soils; even so, there are still cases where failure has occurred. In this technical note, the process and causes of stone column failure in reinforcing collapsible soils is examined. A behavior analysis of a typical element of soil in the vicinity of column during inundation was studied. The difference between the behavior of a stone column in a collapsible fill and a column in a non-collapsible fill is reported in this paper. Also a solution for the problem of stone column failure is suggested.

Keywords Collapsible Soil, Stone Column, Failure, Analysis, Foundation Engineering

چکیده کشورهای متعددی در جهان دارای خاک های شل و غیر پایدار محتوی مواد مختلف خاکی می باشند که هنگام غرقاب شدن ممکن است فرو بریزند و تغییرات اساسی در ساختمان آنها ایجاد شود. ستون های سنگی برای تقویت این خاک ها بکار برده شد. با این حال، شرایطی از این خاک ها که با وجود تقویت فرو ریخته، گزارش شده است. در این یادداشت فنی، فرآیند و علل خرابی ستون سنگی در خاک های فرو ریختنی مسلح بررسی شده است. تحلیل رفتار یک عنصر از این خاک ها در نزدیکی ستون سنگی هنگام غرقاب شدن مورد مطالعه قرار گرفته است. اختلاف بین رفتار ستون سنگی در یک خاک فرو ریختنی و یک خاک دیگر غیر فزو ریختنی در این مقاله گزارش شده است. همچنین راه حلی برای مسئله خرابی ستون سنگی پیشنهاد گردیده است.

1. INTRODUCTION

Collapse settlement is the term applied to additional settlement of a foundation due to wetting-up a partly saturated soil. In order for collapse to occur the soil must have a structure that lends itself to this action. According to reference 1, appreciable collapse of a soil requires the following three conditions: an open, potentially unstable, partly saturated structure, a high enough value of an applied or existing stress component to develop a metastable condition, and a strong soil bonding or cementing agent to stabilize intergranular contacts with a reduction which, upon wetting, will produce collapse.

All cases studied so far, have shown that these soils have a honeycomb structure of bulky-shaped grains with grains held in place by some bonding material or forces [2,3]. The material or force must be susceptible to removal or reduction by the arrival of additional water. When support is removed, the grains are able to slide (shear) pass one another moving into vacant spaces. The temporary strength of these soils is provided in a variety of ways. In cases where the soil consists of sand with a fine silt binder, the temporary strength is due to capillary tension or is related to it [4,5]. However, the majority of collapsing soils involve the action of clay particles bonding sand grains.

Authors of references 6 and 7, using an electron microscope, found that under such conditions the clay grains cluster around the junctions in a random flocculated arrangement, giving a buttress type support to the bulky grains. Gross capillary tensions can also be present in these buttresses. The reduced bonding or rigidity effect in a soil during collapse need not be due to capillary suction or clay bridges. A similar effect can be produced by chemical cementing agents such as iron oxide, calcium carbonate, or welding at the grain contacts. These could restrain the bulky grains from rotating so that a more dense arrangement could be secured.

Whatever the physical basis of the bonding strength, all types of bonding are weakened by the addition of water, thereby allowing local shear stresses [8,9] and/or fine particles movement through the soil matrix (suffusion phenomenon) so that, the structure collapse takes place [10].

The amount and type of treatment for these kinds of soils depends on the depth of the collapsible layer and the support requirements for the proposed structure. The simplest solution is to carry the foundation down to the depth at which the collapse phenomenon is absent or of negligible proportions [11]. Vibro compacted stone columns, both partially and fully penetrating has been used to strengthen such soils, but there are cases where failure has occurred. It is well known that stone columns can be used in soils with strength normally in excess of about 25 kPa. The reason is that, when the stone column strains are loaded it expands radially. The adjacent soil presents similar radial expansion and gives the stone column strength via confinement. Where the yield strength of the ground has reached the column and would gradually collapse with distress normally on the above structure. Little is known about collapsible soils behavior when wetted, although the global effect of collapse settlement is well understood.

In this technical note the failure of stone columns in strengthening collapsible soils is confirmed by laboratory tests. Furthermore, the collapse mechanism of an element of soil during soaking based on changes in lateral stress, is investigated analytically. The obtained results are used to explain the failure process and mechanism of stone columns in a collapsing fill, and also to suggest a solution for this problem.

2. DECREASE IN LATERAL STRESS AS A CAUSE OF COLLAPSE OF AN ELEMENT OF SOIL

As mentioned previously, all types of soil bonding are weakened by the addition of water, thereby allowing local shear stresses to collapse the structure. The factor of safety against sliding (against local shear) in any plane parallel to the failure plane in an element of soil, in the vicinity of a stone column, can be defined as,

$$F_s = \frac{\tau_f'}{\tau'} \tag{1}$$

Where,

F_s = Factor of Safety;

τ' and τ_f' = mobilized effective shear stress and effective shear stress at failure respectively.

From Mohr-Coulomb principal failure criterion,

$$\tau_f' = c' + \sigma_n' \tan\varphi' \tag{2}$$

and referring to Mohr's circle of stress,

$$\sigma_n' = \left(\frac{\sigma_v + \sigma_r}{2} - u\right) + \left(\frac{\sigma_v - \sigma_r}{2}\right)\cos 2\varphi' \tag{3}$$

and

$$\tau' = \left(\frac{\sigma_v - \sigma_r}{2}\right)\sin 2\varphi' \tag{4}$$

Where,

σ_n' = Effective normal stress on the failure plane;

σ_v and σ_r = Vertical and radial stresses applied on the element of soil;

c' and φ'= Cohesion and the angle of shearing resistance of the soil;

u = Pore water pressure.

By substituting Equations 2, 3 and 4 in 1, we obtain

$$F_s = \frac{c' + \left[\left(\frac{\sigma_v + \sigma_r}{2} - u\right) + \left(\frac{\sigma_v - \sigma_r}{2}\right)\cos 2\varphi'\right]\tan\varphi'}{\left(\frac{\sigma_v - \sigma_r}{2}\right)\sin 2\varphi'} \quad (5)$$

Rearrangement of the different terms of Equation 5 gives

$$F_s = \frac{c' + \left(\frac{\sigma_v + \sigma_r}{2} - u\right)\tan\varphi'}{\left(\frac{\sigma_v - \sigma_r}{2}\right)\sin 2\varphi'} + \text{cotg}2\varphi'\tan\varphi' \quad (6)$$

The decrease in volume (collapse) of a partly saturated soil deposit or compacted fill, generally occurs upon increasing the water content at unchanging total stresses (σ_v is constant). Furthermore, the angle of shearing resistance does not change significantly due to wetting induced collapse [12]. Thus, Equation 6 can be written as,

$$F_s = \frac{f\left(c', u, \sigma_r\right)}{g\left(\sigma_r\right)} + k \quad (7)$$

Where,

$$f\left(c', u, \sigma_r\right) = c' + \left(\frac{\sigma_v + \sigma_r}{2} - u\right)\tan\varphi' \quad (8)$$

$$g\left(\sigma_r\right) = \left(\frac{\sigma_v - \sigma_r}{2}\right)\sin 2\varphi' \quad (9)$$

and

$$k = \text{cotg}2\varphi'\tan\varphi' = \text{const.} \quad (10)$$

Before collapse of an element of soil takes place, this factor of safety (F_s) is always greater than unity. But, when soil collapsed upon wetting, F_s decreased drastically to a value significantly less than 1. Referring to Equation 7, the value of F_s decreases when

$f\left(c', u, \sigma_r\right)$ Decreased

or/and

$g\left(\sigma_r\right)$ Increased

Furthermore,

$f\left(c', u, \sigma_r\right)$ Decreases if cohesion (c') decreases or/and u increased or/and σ_r decreases (see Equation 8); while, $g\left(\sigma_r\right)$ increases if σ_r decreases (see Equation 9).

Therefore, for a given vertical stress it has been established that, during the collapse of an element of soil upon wetting, cohesion (c') decreases and pore water pressure (u) increases. This is an agreement with [13,14]. The more important feature of these results is that, when the soil collapses there is a loss in radial confinement (σ_r decreases) accompanying a very significant vertical settlement. This is an agreement with [9,15 and 16]. The reduction of lateral support around an element of soil should not be ruled out when postulating collapse mechanisms. It is believed that, soil collapse is caused by loss of confinement. The failure of a stone column,(having a ratio of length/diameter equal to 18) in a collapsible soil, at the earlier stage of inundation process, from column's tip, is a strong confirmation and support for this finding (see the following section).

3. APPLICATION OF THIS MECHANISM TO DESCRIBE THE FAILURE PROCESS OF STONE COLUMNS IN COLLAPSIBLE SOILS

Stone columns occupy an important place and have a major role in ground treatment methods. They can be used in different types of soils and sites. Their costs are relatively moderate and their installation requires medium-priced equipment. Their use for more than 50 years in reinforcing soft soils has demonstrated their usefulness and makes them one of the most attractive methods in improving bearing capacity and reducing settlement. However, in literature there are examples of stone columns made of reinforced granular soils, failing when wetted. [17] Some field data reported; stone columns have failed in strengthening a chalk fill. There are many other unreported case records of a similar type. Based on reported results on the successful use of stone columns to reinforce soft soils and loose fills a basic question arose, Why did this happen and what were the causes?

Before answering this question, it was decided to produce a testing program similar to those applied on site using a model' sand column (diameter d = 23 mm) loaded in a stress controlled pot, which contained a loose fill made of a collapsible soil, the water level being allowed to rise slowly inside it. The column's material was in direct contact with the collapsible soil and no interface column/soil was considered. The soil characteristics are summarized in Table 1 and the field conditions assimilated on laboratory is schematized in Figure 1.

The first feature of the study was to check the collapsibility of the artificially prepared soil. The specimens were placed in an odometer apparatus of 38mm diameter with a moisture content of 4 % and dry density of 15.4 kN/m^3. The followed procedure was suggested by [18]. The load was gradually increased to 200 kPa and at the end of this loading the specimens were inundated with water, left for 24 hours and the consolidated test was carried out in the normal manner. The collapse potential defined as CP = $\Delta H/H_o$, where H_o is the initial height of specimens, and ΔH the change in height. It was found that the collapse potential was 13 %. Most of the collapse occurred suddenly and less than 20 % occurred slowly. It was concluded that; this as-prepared soil was definitely collapsible and suitable for the main testing program.

To check the repeatability of the main testing program, two further tests were executed. One to determine the bearing capacity of a sand column, of length 410mm, in a `dry' soil (test 1) and the other on a similar sand column loaded up to a working load of 40 % of its bearing capacity and then subjected the surrounding soil to inundation (test 2). The minimum values of repeatability, r for all tests (r value of repeatability below which the absolute difference between two single tests results may be expected to lie with a probability of 95 %, $r = 2.8.\sqrt{V_r}$ where V_r is the repeatability variance, BS812 1984) were found to be 0.26 for test 1 and 0.31 for test 2. The maximum absolute difference in values were noted between two identical tests of the testing program which were respectively 0.11 and 0.16 for tests 1 and 2, which is smaller than the previous repeatability value. Consequently, it can be concluded that test apparatus and procedure of the present experimental investigation were validated for the purpose of the proposed research.

TABLE 1. Some Characteristics of the Collapsing Soil Used.

Soil Composition	G_s	Density (kN/m^3)		$w_{opt.}$ (%)	w_L (%)	w_p (%)	Particle Size Distribution (mm) {C_u = 140}			
		$\gamma_{max.}$	γ_{min}				D_{10}	D_{15}	D_{50}	D_{85}
-78 % Concrete Sand-10 % Leighton Bazzard Sand (90 μm)-12 % Speswhite Kaolin Clay	2.65	20.2	13.7	9	20	13.5	0.003	0.03	0.35	0.75

Next stage was to investigate the performance of sand columns of different lengths in a collapsible soil subjected to inundation. Tests were performed on three different foundational supports (soil alone, a sand column fully penetrated of 410mm length and a sand column partially penetrated of 250 mm length). All the tests were performed under a surcharge pressure of 100 kPa which generated an appropriate stress level similar to that under field-scale conditions. With full inundation, Figure 2, it is noted that the foundation `model' on the soil alone settled by an amount of 53 mm and with the presence of the sand column fully penetrating, it settled by an amount of 52 mm. Furthermore, readings of the settlement of the column and the soil around it, during the process of inundation, taken at equal time intervals, showed that the settlement of the column and the soil around it was approximately uniform and equal at any time during inundation, even for partial inundation and for partial penetration. It was quite clear that when the specimen was inundated with the reinforced sand column there was no reduction in settlement due to the presence of the sand column. The bearing capacity, which was improved in the `dry' state due to the presence of the sand column, was reduced drastically due to full inundation.

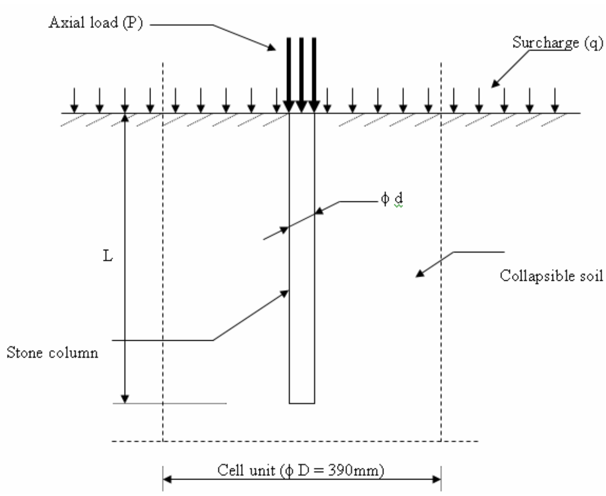

Figure 1. Isolated stone column foundation loaded in a collapsible soil.

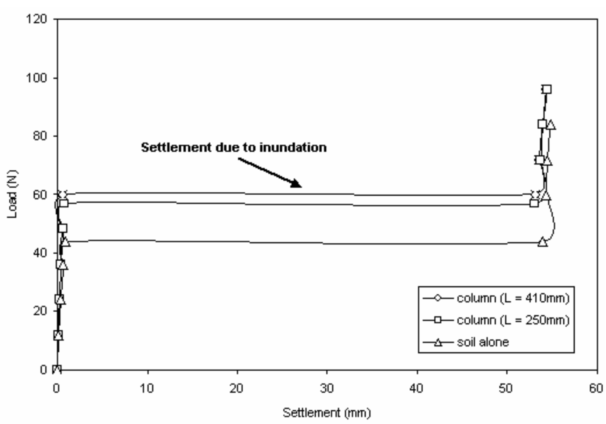

Figure 2. Settlement curves for the collapsible soil and stone columns after inundation under working load of 30 % ultimate.

Similar trends to that of field tests were observed and similar results were obtained with a high level of repeatability. Provided that stone columns generally fail by bulging at relatively shallow depths (typically at about 4 to 8 diameters from the top of the column, [19-21], the water table, in a loose fill (not collapsible), would have to rise close to ground level in order to have any significant effect on the performance of the column when loaded. It was reported that, under such circumstances, the ultimate capacity of the column could be reduced by up to 50 % in the worst cases [22]. However, in this case (a collapsible fill) the capacity of the column was reduced drastically and large settlements were observed even for partial penetration and partial inundation.

After the confirmation of stone columns' failure in reinforcing collapsing soils by laboratory test, a careful investigation was performed in order to answer the question posed earlier. Based on the analysis undertaken in the previous section, it is believed that during the inundation of an element of soil in a collapsing ground, the lateral pressure around the element decreased and a large reduction of its volume resulted from axial as well as lateral deformations. These findings were found to be the main causes of the problem and the phenomenon encountered in collapsible loose fills reinforced by stone columns. To explain the process, Figure 3 has been used. It consists of a fully penetrating stone column loaded in a collapsible soil. Consider the section A-A where the distance between it and the hard layer is an arbitrary value h (equal to the

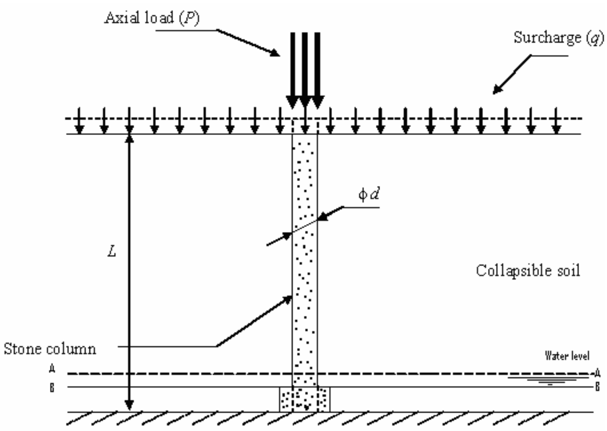

Figure 3. Process of collapse of a stone column in a collapsible soil subjected to inundation caused by rise of the water table.

diameter of the stone column). The small element of the stone column under this section is in equilibrium. The forces acting on it are:

- The vertical forces acting downward due to the loads and the weight of the entire top block.
- The reaction forces acting upward and which are due to the confining pressure around the element provided by the soil.

The water table rises up to the section A-A leading to a large reduction of volume of the soil. Consequently the section A-A will settle to the position of section B-B and the whole block above this section will move downward and will press the thin layer below it. With the decrease of the confining pressure around the small element of the column and some negative side shears from the collapse of the soil caused by inundation, the element will deform laterally, and settle by the same amount as that of the soil. The same explanation can be used for another element of the stone column and so on. This explains how the column settles and fails in strengthening a collapsing loose fill.

The differences between the behavior of a sand column in a collapsible fill and a column in non-collapsible fill resulted mainly from the differences in the behavior of a typical element of soil in the vicinity of the sand column during inundation. Quite clearly, in both cases, there is a change in volume, a change in stiffness and also a change in stress state. However, the magnitudes of these changes are quite different. A change in volume of a collapsible specimen is much larger than that of non-collapsible. Also, lateral deformation resulting from the change in volume of a collapsible specimen is remarkably different from that of a non-collapsible soil. [23] Found that, for specimens of smaller diameter (d), height (h) and a ratio of h/d equal 2, the lateral deformation exceeded the vertical one by a factor of 5 to 17 under all-around pressures. These considerations alone can explain the differences in the behavior of stone columns. The inundation of a non-collapsible fill around a stone column and below its critical length, (defined to be equal to 4-8 d from column top) has a negligible effect on its settlement behavior, because the volume change of a typical element of soil in the vicinity of the column is very small and the changes of stress in the column caused by wetting, are negligible. The water must reach the critical length in order to have any significant effect. This is because the lateral stresses in the column at that part were larger and any change in the passive pressure may lead to the appearance of additional stresses in the surrounding soil, causing its deformation to a limited extend, depending on the soil type. In the case of a collapsible fill this is different. Even below the critical length, the large change in the volume of an element of soil in the vicinity of the column, which resulted from large vertical and lateral strains, caused the downward movement of all the soil mass situated above the element. Consequently, the element of soil and the small portion of the column beside it will undergo additional vertical stresses and this will increase the volume change of the element and also the lateral stresses in the small portion of the column. The small portion of the column will then deform laterally and the deformation will easily be accommodated by soil.

Based on the forgoing laboratory test results and analysis, it is confirmed that stone columns have failed in strengthening a loose fill, which exhibits a collapse behavior caused by inundation. Furthermore, it was shown that this behavior was due to the reduction of confinement provided by the surrounding soil. However, by discovering the causes of the problem, another question could be asked; How to deal with this problem and how to

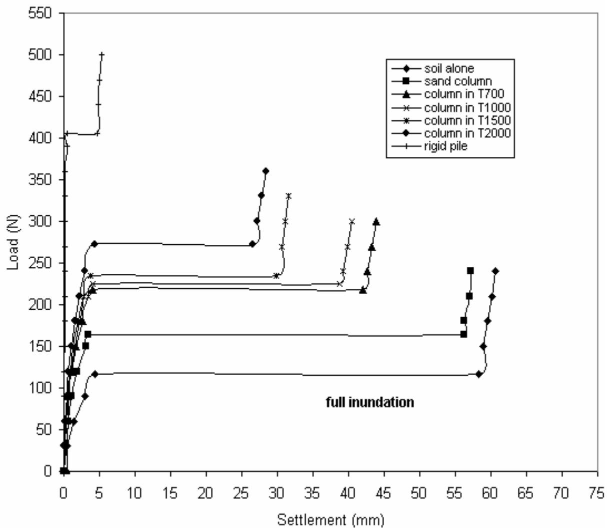

Figure 4. Settlement reduction of a stone column in a collapsible soil caused by encapsulation of the column in terram geofabrics [26].

eliminate or control it? This time the answer was very simple. The elimination of the problem consisted of the prevention of the loss of the confining pressure around the stone column. It was reported that stone columns encapsulated in geofabric, performed satisfactorily in such conditions e.g. [24,25]. Figure 4 represents an example of settlement reduction of a stone column in a collapsible soil caused by encapsulation of the column in Terram geofabrics [26].

4. CONCLUSION

In this investigation, it has been shown analytically that, during soil collapse upon wetting, the lateral stress decreases. The reduction of lateral support around an element of soil should not be ruled out when postulating collapse mechanisms. It is believed that, soil collapse is caused by the loss of confinement. Stone columns have failed in strengthening a loose fill which exhibits a collapse behavior caused by inundation. The failure process of the column during soil collapse has been explained, based on the behavior of a typical element of soil in the vicinity of a column. The difference between the behaviors of a sand column in a collapsible and non- collapsible fill is notably the differences in volume and the state of stress changes, as was examined.

5. ACKNOWLEDGMENT

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6. NOTATIONS

c'	Cohesion of Soil
C_u	uniformity Coefficient
D_x	Diameter of the Soil Particles for Which (x) % of Particles are Finer
F_s	factor of Safety
$f\left(c', u, \sigma_r\right)$	Function of c', u and σ'_n
$g\left(\sigma_r\right)$	Function of σ'_n
G_s	Specific Gravity
k	Constant
u	Pore Water Pressure.
w_o	Initial Moisture Content
$w_{opt.}$	Proctor Optimum Moisture Content
W_L	Limit of Liquidity
W_P	Limit of Plasticity
φ'	Angle of Shearing Resistance of the Soil;
$\gamma_{max.}$	Maximum Unit Weight
$\gamma_{min.}$	Minimum Unit Weight
τ' and τ'_f	Effective Mobilized Shear Stress and Effective Shear Stress at failure respectively
σ'_n	Effective Normal Stress on the Failure Plane;
σ_v and σ_r	Vertical and Radial Stresses Applied on the Element of Soil.

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